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Design and construction of an excavation under railway bridges at Reading, England

Conception et construction d'une excavation sous les ponts ferroviaires à Reading, en Angleterre

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ABSTRACT: In order to construct a concrete U-trough for a new carriageway a deep excavation was required beneath two existing rail viaducts and a rail overbridge, northwest of Reading Station, Reading, England. A temporary sheet pile cofferdam, embedded into chalk, was installed over much of the length of the excavation, but at the location of the viaducts and the overbridge alternative measures were required due to limited construction envelope which prevented sheet pile installation. The ground conditions at the site consisted of the made ground overlying Alluvium and River Terrace Deposits which was underlain by structureless chalk, but the solution was complicated by the fact that groundwater was typically 1.0 m below the existing road level, and an existing road link and rail lines, supported by the bridge and viaduct, were to remain operational during the works. This paper describes the solution developed to overcome the site constraints. This comprised grout injection between piled foundations and at the cofferdam interface, an array of dewatering wells and a controlled construction and excavation sequence to manage risks associated with the excavation in proximity to the road and rail environment.

RÉSUMÉ: Afin de construire une rigole en U en béton pour une nouvelle chaussée, une excavation de 3,8 mètre de profondeur était nécessaire sous deux viaducs ferroviaires existants et un pont roulant, au nord-ouest de Reading Station, à Reading, en Angleterre. Un batardeau temporaire pour palplanches, encastré dans de la craie, a été installé sur une grande partie de la longueur de l'excavation, mais des emplacements alternatifs ont été nécessaires à l'emplacement des viaducs et du pont supérieur, tels que fondations sur pieux, et une enveloppe de construction limitée empêchant l'installation de palplanches. Les conditions du sol sur le site consistaient en un sol recouvert de dépôts alluvions et de graves de la terrasse sur une craie sans structure, mais la solution était compliquée par le fait que les eaux souterraines se trouvaient généralement à 1 mètre en dessous du niveau de la route existante, et les voies ferrées, soutenues par le pont et le viaduc, devaient rester opérationnelles pendant les travaux. Ce document décrit la solution développée pour surmonter les contraintes du site. Cela comprenait l'injection de coulis entre les fondations sur pieux et à l'interface du batardeau, un ensemble de puits de déshydratation et une séquence contrôlée de construction et d'excavation visant à gérer les risques associés à l'excavation à proximité de l'environnement routier.

Keywords: Cofferdam; sheet piles, grouting; dewatering; chalk

1 INTRODUCTION

Cow Lane is located on the west side of Reading, linking Richfield Avenue to the north with Portman Road/Beresford to the south. As part of a scheme to improve the alignment and provide compliant headroom, two original bridges were replaced and the carriageway was temporarily realigned so that it past beneath the newly constructed 2-span railway MLN overbridge (the elevated Main Line structure) and two new railways viaducts (named as Festival Line and Central Line), as shown in Figure 1.

The final phase of the development comprised the demolition of a masonry arch and embankment, associated with the original bridge, and the lowering of the highway alignment within a concrete U-trough, constructed beneath the bridge and viaducts. The excavation for the trough construction required an excavation with a maximum depth of 3.8 m which needed to extend beneath the bridge and viaducts and adjacent to the temporary road alignment. All of which were to remain operational during the construction works.

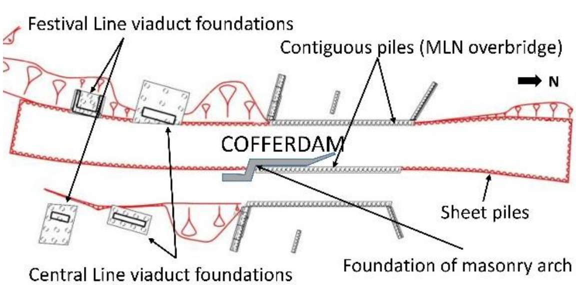

Figure 1. Cofferdam and site constraints

The Principal Contractor BAM Nuttall Ltd. appointed Tony Gee and Partners LLP as the designer, WJ Groundwater Ltd. as the dewatering specialist contractor and BAM Ritchies as the specialist grouting contractor. The client for the project was the Berkshire County Council while the railway asset owner and key stakeholder was the Network Rail.

The viaducts are supported by piled foundations whereas the railways bridge is supported by contiguous piles. Based on the record drawings, the contiguous pile foundations supporting the MLN overbridge structure were shown to be 900 mm diameter bored concrete piles at 1050 mm spacing. The Festival Line and Central Line viaducts were supported by 900 mm and 1050 mm diameter concrete bored piles, respectively.

The proposed U-trough alignment provided limited lateral clearance adjacent to the MLN overbridge and viaduct structures, whilst there was also a client requirement to keep the MLN overbridge and the U-trough as separate structures. The length and the width of the excavation for the U-trough was approximately 120 m and 15 m, respectively.

In conjunction BAM Nuttal it was agreed that a steel sheet pile cofferdam was the preferred temporary works solution. But it was evident that at the location of existing overbridge and viaducts sheet piles could not be installed due to the lateral clearance requirements and therefore an alternative solution would need to be found. The installed sheet piles were 12 m long AZ26-700 steel sheets.

This paper describes the solution developed to overcome the constraints outside the area of conventional sheet piling. This comprised an integrated grouting and dewatering solution with permeation grouting between piled foundations and at the cofferdam interface; an array of dewatering wells to achieve a safe and dry working environment; a controlled construction and excavation sequence; and a construction monitoring regime to evaluate ground and groundwater behaviour during the works.

2 GROUND CONDITIONS

Based on the 1:50,000 scale geological published map of Reading by British Geological Survey (BGS), the stratigraphy at the site consists of undifferentiated Alluvium or the Langley Silt Formation, which varies from silt to clay, and overlying sands and gravels of the River Terrace

Deposits (BGS, 2000). These superficial deposits are underlain by the Seaford Chalk Formation of white nodular firm chalk with flint seams. The ground conditions from the ground investigations at the site are consistent with the geological information. The made ground material comprising coarse- and fine-grained constituents without any consistent pattern was also encountered.

Based on the borehole log information the chalk was classified as structureless weathered CIRIA Grade Dm (fine-grained) and Dc (coarse-grained) chalk as per Lord et al. (2002). Structured chalk was encountered below Grade Dc chalk. Table 1 presents the design stratigraphy. The final excavation level was within the River Terrace deposits.

Table 1. Stratigraphy used in design

Stratum	Top Level (m OD)	Base Level (m OD)
Made Ground	39.1	37.1
Alluvium/Langley Silt	37.1	35.3
River Terrace Deposits	35.3	30.4
Chalk (Grade Dm)	30.4	26.1
Chalk (Grade Dc)	26.1	unknown

* OD: Ordnance Datum Newlyn

Groundwater monitoring was carried out indicating that groundwater levels were typically at the top of the terrace deposits, approximately at 35.0 mOD. The terrace deposits are considered to be hydraulically linked to the underlying chalk although the weathered (Grade Dm) chalk zone may form an aquitard, impeding groundwater movement vertically. Both soil zones would also form continuous and extensive aquifers laterally and probably have a connection with the River Thames which is approximately 800 m north of the site.

In order to understand in more detail potential groundwater flow rates into the excavation additional ground investigation consisting of rotary coring into chalk and permeability tests were carried out in 2017. This investigation was also designed to compensate for apparent discrepancies and ambiguities in previous permeability tests undertaken for the permanent works design. The supplementary investigation included a series of rising head tests at various depths within the weathered chalk. To avoid erroneous readings which can be caused by drawdown from the gravels via the exterior of the casing and which might have influenced the results from previous investigations, a bentonite seal was positioned at the terrace deposits/chalk interface.

The 2017 rising head tests in the chalk showed permeability values between a depth of 10 m and 14 m which was the proposed embedment depth of the sheet pile wall. The coefficient of permeabilities are shown in Table 2.

Table 2. Measured coefficient of permeability, k

Stratum	k (m/s)
River Terrace Deposits	$2x10^{-3} – 8x10^{-4}$
Chalk (Grade Dm)	$1x10^{-4} – 6.5x10^{-5}$
Chalk (Grade Dc)	$8x10^{-5}$

3 PRECONSTRUCTION PLANNING

Different phases of the construction works planned in the design stage, excluding the installation of the sheet pile cofferdam, are detailed in the following sections.

3.1 Evaluation of permanent works design

Original design calculations for the overbridge indicated that the contiguous piles had been designed to support the load imposed by the soil and groundwater during the temporary excavation. However no calculations were available for the viaduct structures. Analysis was therefore undertaken to assess whether the foundations could provide sufficient ground and groundwater support during the excavation stage. This analysis confirmed that piles had sufficient capacity to support the soil and water. Provided that the soil between the piles could be improved to be self-supporting, so that load could be shed onto the adjacent piles, an additional retention system would not be required. Achieving this, the focus of the design could be directed towards controlling groundwater ingress.

3.2 Grouting

The primary constraint to the solution developed outside the area of the sheet piles was the high water table and the permeable nature of the near surface deposits. To ensure that any water ingress into the cofferdam was minimised to prevent soil flow into the excavation a permeation grouting solution combined with a dewatering system was chosen. Grout was to be injected at the location of the interfaces with existing structures, between the underbridge contiguous piles and between the viaduct piles.

The grouting design was developed in conjunction with BAM Nuttall and BAM Ritchies. This design included modification/refinement to the original proposals, which defined the injection cementitious grout behind the contiguous piles to form a seal at the back face. This was a necessary modification, due to access constraints and consisted of grouting on the front face of piles at higher level with the additional provision of a sprayed gunnite facing wall.

The final solution comprised the injection of cementitious grout via tube à manchettes (TaMs), formed from of 63 mm diameter pipes grouted into a nominal 150 mm diameter borehole. An array of TaMs were installed between each of the viaduct pile foundations at 900 mm horizontal spacing. Between each contiguous pile of the overbridge two levels of TaMs were installed with an embedment of 1000 mm into the chalk. Grout was injected at each machette location positioned at 330 mm vertical centres within the treatment zone.

A detailed performance specification was prepared which required the terrace gravels to achieve a post grouted permeability of $1x10^{-7}$ m/s and included a suite of verification tests to demonstrate performance and also a provision for resin grouting to remediate any post excavation water ingress.

3.3 Dewatering

Dewatering was required to lower the groundwater table within the cofferdam to a minimum 1 m below the final excavation level. One of the site constraints was discharge consent of 100 l/s.

To evaluate potential groundwater flow rates the SEEP/W software by GeoStudio was used for preliminary analysis. Using cautious assumptions and the permeability coefficient of the post grouted terrace deposits as $1x10^{-7}$ m/s, the preliminary analysis indicated a potential flow rate of approximately 0.0011 m^3/s/m over the width of the excavation within the sheet piled area. In order to assess an upper bound discharge requirement the preliminary analysis also considered the possibility of the failure of the TaMs to penetrate the top of the chalk and a lower performance of the grout. Based on the SEEP/W analysis, taking the length of sheet pile section of the cofferdam as 226 m with flow rate of 0.00055 m^3/s/m and a combined length of overbridge and viaduct foundations of 70 m with a flow rate of 0.002 m^3/s/m, the dewatering system over a total perimeter length would need to pump approximately 0.26 m^3/s/m (260 l/s).

Based upon detailed assessment by the dewatering specialist contractor WJ Groundwater, analysis of a series of modelling scenarios suggested a flow rate of 74 l/s to 141 l/s considering lower permeability at the grouted contiguous wall section. WJ Groundwater designed a dewatering system with a capacity of 100 l/s but included a provision for it to be upgraded to 200 l/s if a higher capacity was needed. The dewatering system comprised of 10 deep wells and 12 monitoring piezometers, of which 11 internal and 1 external.

3.4 Ground movement assessment and monitoring system

A construction monitoring regime was essential to monitor real time settlements behind the wall to check the validity of design-stage performance predictions and to provide warnings during the temporary works of the cofferdam sheet pile installation, grouting at the existing bridge pile foundations and contiguous wall, dewatering, excavation within the cofferdam, permanent works U-through construction and removal of the sheet pile cofferdam.

Due to level of uncertainty in numerically modelling the ground conditions and various construction methods and phases accurately, empirical (correlations based on case histories) and semi-empirical methods (correlations based on soil strength and lateral wall displacements) were used to estimate horizontal and vertical ground movements to set up trigger levels for monitoring.

Monitoring scheme included reflective survey targets placed at the cofferdam and the contiguous pile, daily monitoring of bridge and viaducts, and settlement studs on the ground behind the excavation.

4 DURING CONSTRUCTION

Implementing the design on site often bring its own challenges due to practical needs and unforeseen site and ground conditions. These challenges are discussed below.

4.1 Grouting operations

Grouting operations at the bridge and viaduct foundations commenced after grouting trials to refine the grout mix. The trials indicated pumping between 8 l/min with the option of increasing to 10 l/min at maximum. Colloidal silica and MasterRoc MP320 was used in the grout mix. The grouting methodology consisted sequentially grouting each TaM port from the base upward. At each grout round the TaMs were flush with water to enable a subsequent round of grouting.

A theoretical grout take per injection level was calculated based on a 300 mm high and 600 mm radius of grout bulb and an assumed void ratio of 0.3. This defined the average expected grout volumes per injection level between 50 l and 100 l. In order to avoid ground heave/break through grout pressure were initially limited to two times the overburden pressure.

As the grout takes were considered to be one key indicators of effectiveness of the grout to penetrate the ground they were closely monitored during the grouting works.

4.1.1 Grouting at contiguous piled wall

During the construction works the upper portion of the contiguous piles was uncovered so that the spacing between the piles could be surveyed. This had been originally identified as a risk on the project risk register. The survey found that at a limited number of locations the spacing between the piles was smaller than the anticipated 150 mm. At these locations it would not be possible to install TaMs, thus it was agreed to use a 120 mm casing which got through the smaller gaps. Where piles were in direct contact, gaps were sealed with the sprayed concrete.

4.1.2 Grouting at viaduct foundations

At viaduct locations, grouting at the face of the pile foundations was carried out as indicated in Figure 2. However, at a number of locations the installation of TaMs was obstructed by a greater pile cap depth than the one shown on the as-built drawings. Furthermore, the location of the second row of piles appeared to be inconsistent with the as-built drawings.

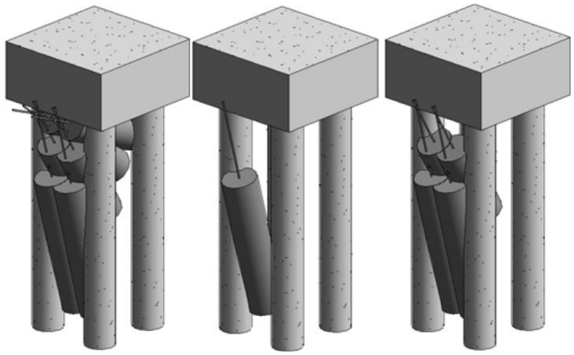

Figure 2. Indicative grouting stages at foundation locations as planned, TaM 1, 4 and 8 from left to right.

As a consequence, at some locations, after multiple failed attempts to install the TaMs and a review of the grout takes, a decision was taken to abandon further attempts and proceed with the dewatering phase. This decision was taken in the knowledge that piezometers, to be installed as part of the instrumentation and monitoring programme, had been positioned to identify if sections of the grouting works were not performing as expected and that there was also a provision for supplementary grouting if the evidence of slow drawdown in the location of the viaduct foundations was seen.

4.1.3 Grout takes versus theoretical volumes

A considerable amount of time was spent during the grouting works attempting to reconcile actual grout takes with those predicted prior to the works. Two issues were evident and can be seen in an idealised representation of grout takes on the east face of the MLN overbridge, presented in Figure 3.

With the exception of a number of injection points, represented by the largest spheres in Figure 3, most grout takes were significantly lower than the predicted theoretical volumes. In addition, at the location where the TaMs passed between the contiguous piles (represented by the two red dashed lines in Figure 3) lower grout takes were expected, but often this was not observed. Instead low grout takes were seen to be more randomly distributed along the length of the TaMs (represented by the smallest spheres in Figure 3).

In order to investigate whether the lower grout takes represented failure of the grout to penetrate the gravels a trial was undertaken at the location of the Central Line viaduct. At this location an additional TaM was installed between two previously grouted TaMs which had undergone a second round of grouting with low grout takes. The intermediate TaM was grouted with a leaner mix (higher water to cement ratio) and at higher grouting pressures. The trial showed an increase in grout take suggesting that the initial grouting may not have sufficiently penetrated the terrace gravels.

There was some evidence however that gravels contained a higher fines content than originally assumed, which had been based on the ground investigation results reporting that the material to be a gravelly sand or sandy gravel. It was considered quite possible that there was a loss of the finer soil fraction on extracting the wet sample during the cable percussive drilling. Notwithstanding this in some locations high grout takes, close to the theoretical volume, were observed and so it was evident that the particle size distribution of the gravel was likely to be variable.

Due to project programme constraints further investigation was not possible. A decision was made to curtail further grouting works. Further grouting was considered if dewatering would fail to adequately lower groundwater levels.

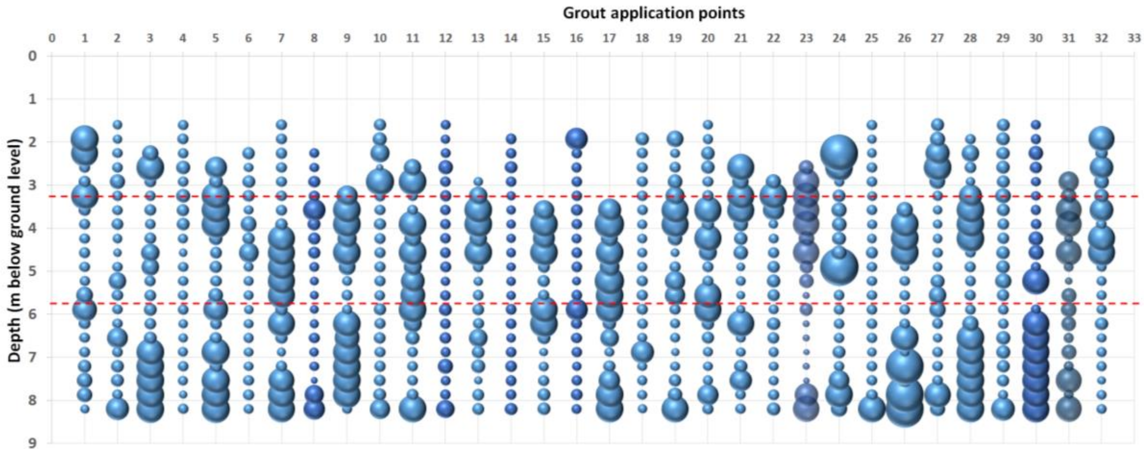

Figure 3. Achieved grout volumes at the east contiguous piled wall of MLN overbridge with depth.

4.2 Dewatering and excavation

The dewatering works followed the cessation of grouting. The water levels were monitored continuously during the lowering period but it was not possible to reduce levels to more than approximately 0.5 m above the lowest formation level (beneath the MLN overbridge) even though the dewatering rates were typically 50% lower than those anticipated during the design stage by WJ Groundwater. This was rectified by the installation of two additional wells within the centre of the excavation to increase the drawdown rate. In order to try to increase the water flows local excavations were also dug around the wells to extend the perforated area of the wells as the fines were found to be clogging up the original perforated area. This helped improve the drawdown rates until water level was reduced to a below formation steady state level with a pump rate of approximately 60 l/s.

These findings provide support to the interpretation that in areas the terrace deposits contained more fines than originally expected and that the un-grouted permeability was more variable than originally assumed.

As a whole, the grouted zones were found to be very effective in reducing groundwater ingress. During excavation only very local areas of seepage were observed which is likely to represent local zones of cleaner gravel prior to grouting. Of note was the location at the interface between the end pile of the contiguous piled foundations and the start of cofferdam sheet piles (Figure 4).

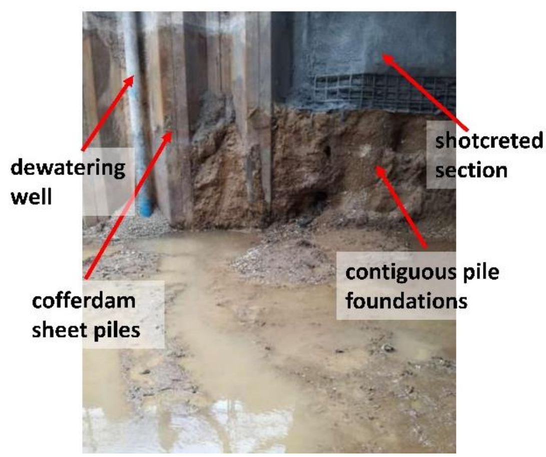

Figure 4. Water ingress at sheet pile and overbridge contiguous piled foundation interface.

At this location a gap was present between the end pile and the adjacent sheet pile of the cofferdam where the pile cap of the MLN overbridge overhung the contiguous piles. As the excavation progressed water ingress was observed and it was evident that the grouting had failed to adequately reduce the permeabilty of the gravels at this location. This also occurred on the southwest interface but to a lesser degree. To rectify the seepage, the road was closed and excavation from existing road level was carried out. The loose material was cleared out behind the sheet pile and the void was backfilled with concrete. Shotcrete was only carried out within the MLN overbridge foundations on the exposed contiguous piles (Figure 5). Resin was used at formation level to mitigate areas of small water ingress and at interface with the central line viaduct pile cap and the sheet piles.

Figure 5. After concreting at interface shown in Figure 4.

4.3 Ground movement

Ground movements during the construction were within the acceptable limits set by trigger level criteria. At the track levels no movement breaches were observed.

5 CONCLUSIONS

An integrated sheet pile, grouting and dewatering solution provided a safe and dry working environment for the construction of the road trough beneath existing viaducts and bridges at the Cow Lane, Reading, England.

While the pre-design ground investigations targeted permeability testing in the chalk, analysis of grout takes during the works suggested that the particle size distribution of the gravel was likely to be significantly variable and the level of variation had not been appreciated at the investigation stage. This is considered to have resulted in marked differences in pre-grout permeability within the gravels across the excavation footprint and manifested itself in significant variance in grout takes.

Whilst the variability in grout takes provided a level of uncertainty about the permeation of the grouting works the scheme benefited from an overarching design strategy which provided a means of verifying the effectiveness of the water cut-off (formed by both the grouting and piling works). This was achieved through a monitoring regime which allowed the evaluation of drawdown during the works and so provided confidence for the later works to proceed.

From the outset a collaborative working environment between the designer, the specialist contractors and the principal contractor was established. This ensured that design risks, unforeseen site and ground conditions, performance uncertainties and construction challenges could be openly discussed such that pragmatic and yet informed decisions could be taken at key stages in the project.

6 ACKNOWLEDGEMENT

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7 REFERENCES

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